Horizontal Connections for Precast Concrete Shear Wall Panels Under Cyclic Shear Loading

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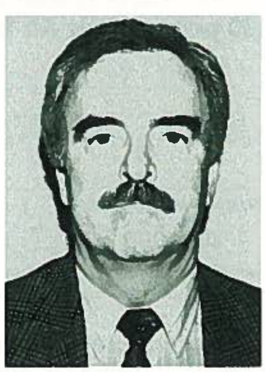

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The performance of precast concrete loadbearing shear wall panel structures during an earthquake relies on the behavior and integrity of the connections between the panels. Design of these structures requires the ability to predict the behavior of the connections. This paper presents results of an experimental study of horizontal connections for precast wall panels subjected to reversed cyclic shear deformations combined with simulated gravity loads normal to the connection. The study considered typical connection configurations currently used in practice. The influence of mild steel reinforcement, post-tensioning and shear keys was investigated. Experimental results were used to determine the cyclic behavior of the connections and to identify the contribution of the connection components. Simple rational models were proposed to predict the limit states of the connections. Two numerical design examples are included to illustrate the proposed models. Based on the study, design recommendations are presented.

The precast concrete loadbearing shear wall panel system is an economical construction system for low, medium and high rise structures. This form of multi-story structural system is composed of large precast concrete wall panels and floor panels arranged to form box-like apartment or office size spaces. The walls of the structure carry both the gravity and lateral loads on the structure and an intermediate frame is not required. The wall panels are typically one story in height and are connected both horizontally and vertically to adjacent panels at each story.

Vertical connection between panels may be provided by welded or bolted shear connectors. At the horizontal connection, both shear and vertical forces must be transferred. Vertical continuity is normally achieved using post-tensioning or with welded or mechanically spliced mild steel continuity bars. Mechanical shear connectors or multiple shear keys are often used to enhance shear transfer at the connection. During erection, a gap is left between adjacent panels for alignment purposes. In North American practice, this space is filled with drypack grout to complete the connection.

The advantage of this type of structural system lies in its high quality precast elements and rapid cost-effective erection. Cost savings compared to other forms of construction can be achieved through the use of large wall panels with minimal connections. However, this approach places large demands on the connections between the elements. In general, the connections have lower strength and stiffness than the wall panels and form planes of weakness. Because the horizontal connections of the structure must transmit both the gravity and lateral loads on the structure, they play a critical role in sustaining structural stability and integrity. Consequently, it is important to fully understand their behavior.

In seismically active regions, the use of precast concrete structural systems has primarily been limited to low rise structures. The main reason for their exclusion from use in medium and high rise applications is the lack of knowledge of how this type of construction will perform under seismic loading conditions. Coupled with this shortage of technical information, the current building codes and design guidelines of North America[1-7] do not specifically address the use of precast concrete in seismic zones. Rather, precast concrete is required to comply with code provisions developed for cast-in-place concrete that do not consider the unique behavior of precast concrete and in most cases eliminate the competitive advantages of its use.

Acceptance and competitiveness of precast concrete loadbearing shear wall panel systems in the seismic zones of North America depend on understanding the behavior of the connections and development of guidelines and recommendations for the design and use of these types of connections.

To achieve this goal, a multi-phase research program was undertaken to investigate the behavior of typical horizontal connection configurations under reversed cyclic shear loading equivalent to earthquake loading conditions. Connections were also subjected to axial stresses normal to the connection to simulate gravity loading conditions. A companion research program[8-10] was conducted in parallel to consider the cyclic flexure and shear behavior of currently used connections and innovative connection details believed to enhance the ductility and/or capacity for energy dissipation. The intent of the research is to fully define the horizontal connection behavior to promote development of precast shear wall panel structural systems for use in both moderate (Zone II) and severe seismic zones (Zones III and IV).

Table 1. Outline of experimental program.

Type	Connection description	Test method	
DP	Plain surface, drypack grout only	*	Cyclic
SK	Shear keys only	*	Cyclic
RW	Reinforcing bar welded to steel angle	*	Cyclic
PTS	Post-tensioned strand	*	Cyclic
PTB	Post-tensioned bar	Static	Cyclic
Total number of shear specimens = 6		1	5

* Static tests performed in previous studies at the University of Manitoba (Refs. 11-13).

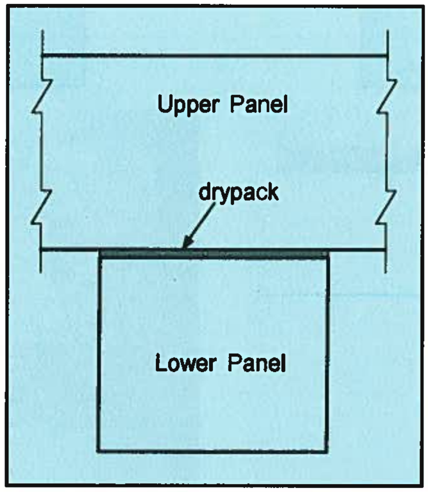


Fig. 1(a). Connection DP — Drypack grout only.

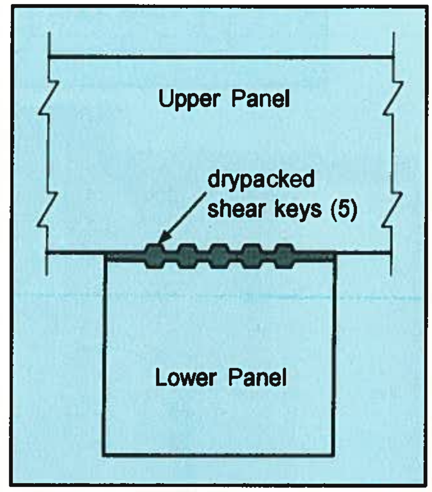


Fig. 1(b). Connection SK — Shear keys.

The scope of these research programs expands a previous five-year research investigation of the monotonic shear behavior of typical precast concrete shear wall connections performed at the University of Manitoba and published in the PCI JOURNAL (Foerster et al.,[11] Hutchinson et al.,[12] and Serrette et al.[13]).

RESEARCH SIGNIFICANCE

The experimental program[14] describes the cyclic shear behavior of horizontal connections incorporating prototype connection details currently used by the precast concrete industry. The observed behavior was used to identify the various limit states and modes of failure under cyclic loading. The experimental results were used to propose mechanisms of shear transfer for the different connection configurations and to develop simple analytical models to predict the connection behavior at the various limit states. The research study was used to introduce

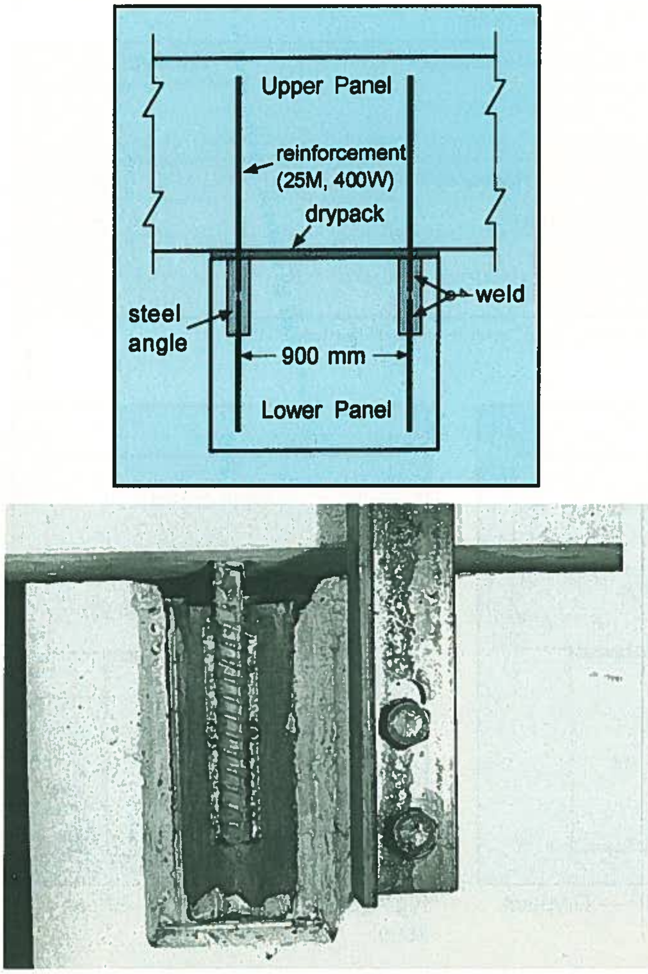


Fig. 1(c). Connection RW — Reinforcing bar with welded connection.

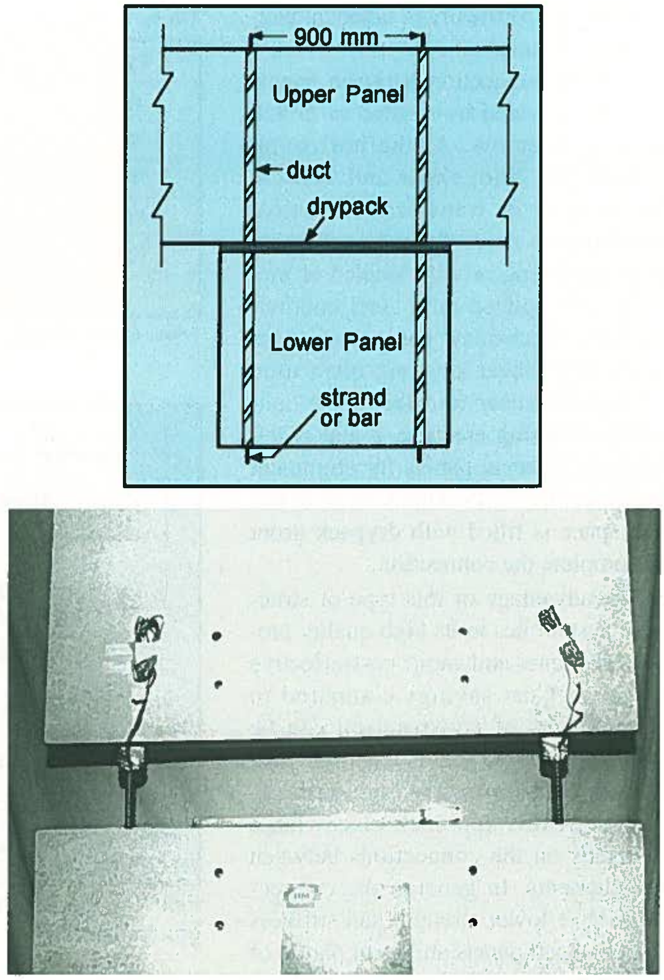


Fig. 1(d). Connections PTS and PTB — Post-tensioned using strands or bars.

shear design recommendations for horizontal connections for precast concrete loadbearing shear wall panel systems in seismic zones.

EXPERIMENTAL PROGRAM

The experimental program was designed to evaluate the connection behavior under the effects of cyclic shear conditions utilizing the static (monotonic) test results conducted on the same connections in the previous study at the University of Manitoba.[11-13] A total of six specimens were considered with five different connection configurations. All connection configurations were tested under cyclic loading. A sixth specimen was tested under monotonic shear loading as one connection configuration was not considered in the previous research mentioned above. Table 1 summarizes the experimental program.

Connection Configurations

The five connection configurations tested are shown in Fig. 1. All connections incorporated a 20 mm ($^3/_4$ in.) thick gap between the two panels filled with drypack grout. All vertical continuity reinforcement or post-tensioning was centered on the connection and spaced at 900 mm (36 in.) apart. The five connection configurations are as follows:

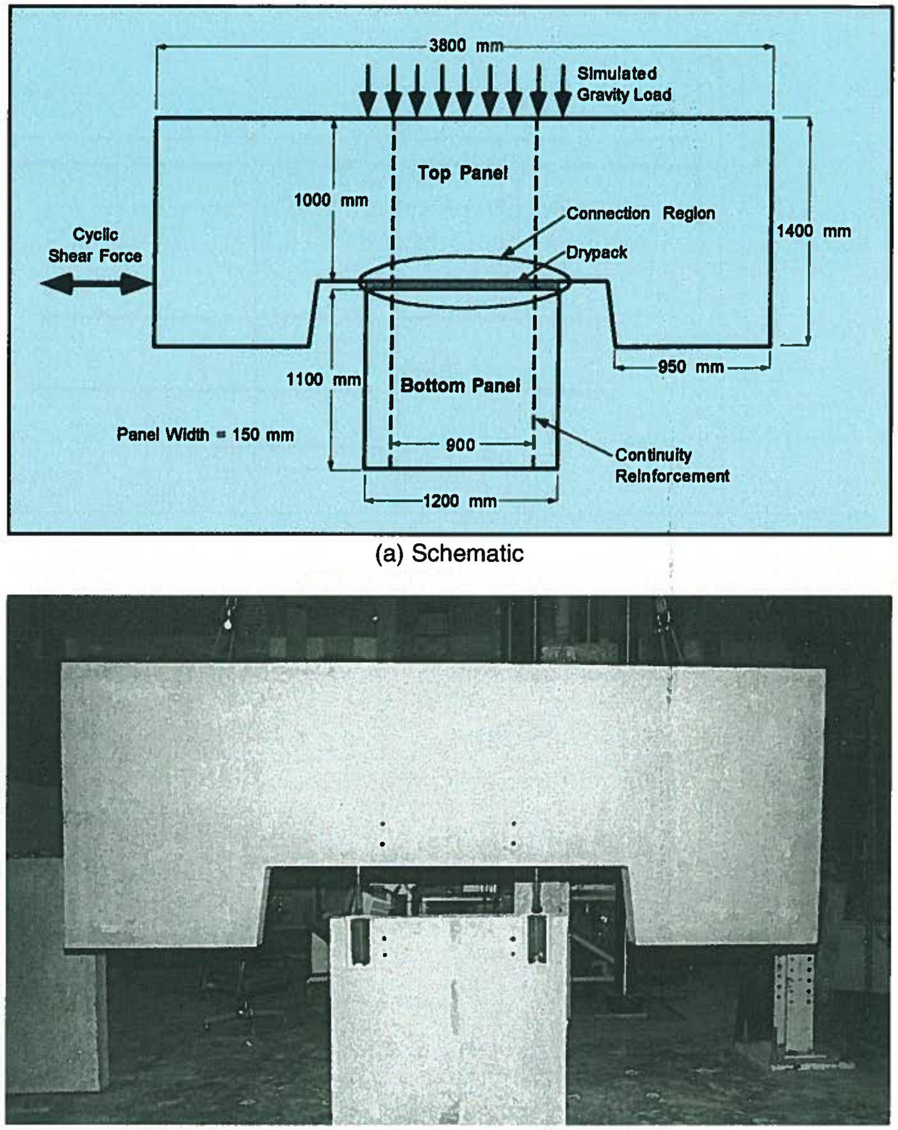
DP: Drypack Plain Surface — Connection DP had a plain surface region filled with drypack, as shown in Fig. 1(a). This specimen was used to identify the contribution of the drypack to the connection behavior and is the control specimen for the study. Typically, some form of vertical continuity element is required across the connection.

SK: Multiple Shear Keys — The connection interface surface for Connection SK consists of five shear keys, as shown in Fig. 1(b). The length of the shear key is 100 mm (4 in.), the depth is 35 mm ($1^3/_8$ in.) and the sides of the key are inclined at 23 degrees from the vertical. The shear keys and the gap between the panels are completely filled with drypack grout. No vertical reinforcement is provided across the connection interface in order to study the influence of the shear keys alone.

RW: Reinforcing Bars Welded to Steel Angle — This connection is commonly used by the Canadian precast industry. The connection consists of a plain surface region with two 25M (#8), Grade 400W (60 ksi) mild steel bars. The straight bars protruding from the upper panel are welded to a 75 x 75 x 10 mm (3 x 3 x $^1/_2$ in.), Grade 300 (40 ksi) steel angle in an exposed pocket in the lower panel, as shown in Fig. 1(c). After welding, the gap between the panels is filled with drypack grout.

PTS: Post-Tensioned Strand — Vertical continuity in Connection PTS is provided using two 12.7 mm ($^1/_2$ in.) diameter, Grade 1860 (270 ksi) seven-wire strands. The gap between the panels was filled with drypack grout and allowed to cure for seven days prior to post-tensioning. The strands were tensioned to 60 percent of their ultimate strength to produce an average stress of 1.2 MPa (174 psi) normal to the connection. The strands were placed inside galvanized steel ducts cast in the panels. After post-tensioning, the ducts were filled with an expansive grout. The PTS configuration is shown in Fig. 1(d).

PTB: Post-Tensioned Bar — Connection PTB was post-tensioned using two 15.8 mm ($^5/_8$ in.) diameter, Grade 1080 (157 ksi) threaded prestressing bars. Similar to Connection PTS, the bars were prestressed seven days after drypacking to produce an average compressive stress of 1.2 MPa (174 psi) normal to the connection. The bars were placed inside galvanized steel ducts. After post-tensioning, the ducts were filled with expansive grout. The PTB configuration is shown in Fig. 1(d).



(a) Schematic

(b) Photographic

Fig. 2. Test specimen.

Test Specimen and Setup

The overall geometry of the test specimens was selected to induce shear loads at the interface of the connection as shown in Fig. 2. The specimen consisted of an upper panel and a lower panel joined at a 1200 mm long and 150 mm wide connection (48 in. long x 6 in. wide). The upper panel had an inverted U-shape with double corbels to allow the application of the reversed cyclic shear loading.

In addition to shear loading, all specimens were subjected to a uniform vertical stress, perpendicular to the connection, to simulate the effects of gravity loads for a typical ten-story structure of this type. The magnitude of this stress was 2 MPa (290 psi) for all specimens except the drypack grout only connection (Connection DP), which was subjected to a vertical stress of 4 MPa (580 psi).

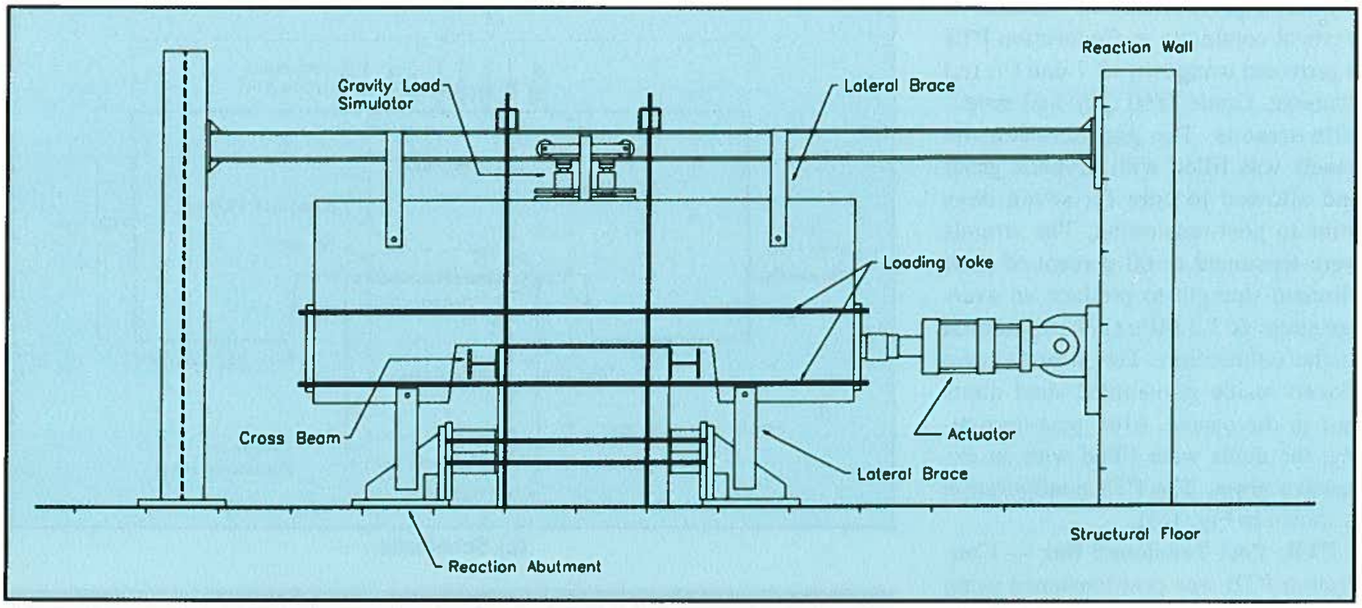
The specimens were tested using a structural steel loading frame as shown in Fig. 3. The lower wall panel of the specimen was fixed to the floor of the structures laboratory by post-tensioning against two reaction abutments, as shown in Fig. 3. The cyclic shear loading was applied to the upper concrete panel using a push/pull loading yoke and a 1000 kN (225 kips) capacity MTS closed loop cyclic actuator. The configuration of the upper panel and the loading yoke allowed the application of a concentric shear load to the connection region.

The equivalent gravity load, normal to the connection, was applied using an independent system designed to allow free displacement in the direction of the applied shear load. The system utilized two hydraulic jacks, which maintained a constant vertical load normal to the connection using a regulator valve and a hydraulic pump.

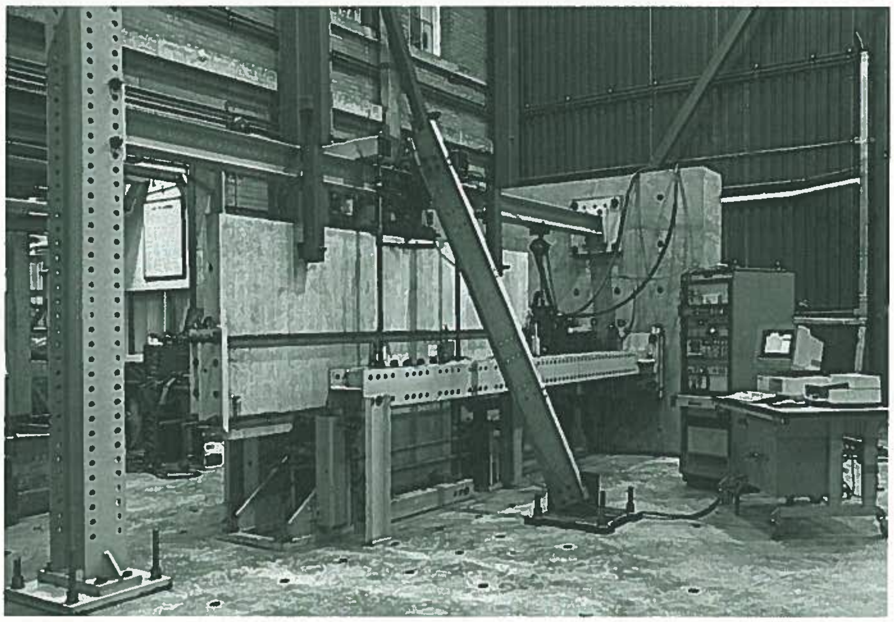
The specimen was instrumented with linear variable differential transducers (LVDTs) and mechanical dial gauges to measure the relative horizontal slip between the two panels and the vertical deformation of the drypack grout. Strain gauges were placed on the post-tensioning bars or mild steel continuity reinforcement at the connection level.

Test Procedure

Prior to application of the simulated gravity load normal to the connection, initial readings were recorded for all instrumentation. The load normal to the connection was then applied and maintained for the duration of testing.



(a) Schematic



(b) Photographic

Fig. 3. Test setup.

The test proceeded by applying the shear loading following a quasi-static, cyclic loading history. A combination of load and displacement control was used to determine the behavior of the connection in the elastic and inelastic ranges. Each load/displacement level was maintained for three fully reversed cycles.

Initially, loading was applied under load control with increments of 50 or 100 kN (11.2 or 22.4 kips). Following the initiation of slip at the connection, the behavior of the connection became inelastic and loading was switched to displacement control. In this program, the initiation of slip constituted a relative slip of 0.1 mm (0.004 in.) between the two panels. Displacement increments were taken as 1.0 mm (0.04 in.) of relative horizontal slip measured between the panels at the connection.

The test was continued under displacement control until failure of the connection occurred, defined as a 20 percent reduction in the shear resistance of the connection. After failure, the test was concluded by applying several single cycles at increasing slip magnitudes to determine the residual strength of the connection. The general form of the loading history is shown in Fig. 4.

EXPERIMENTAL RESULTS

Connection Behavior

The general behavior of the connections tested under cyclic shear loading was quite similar for the different connection configurations considered in this investigation with the exception of the shear key configuration, Connection SK. The shear resistance-slip behaviors under reversed cyclic shear loading for the five connection configurations are shown in Figs. 5(a) to 5(e).

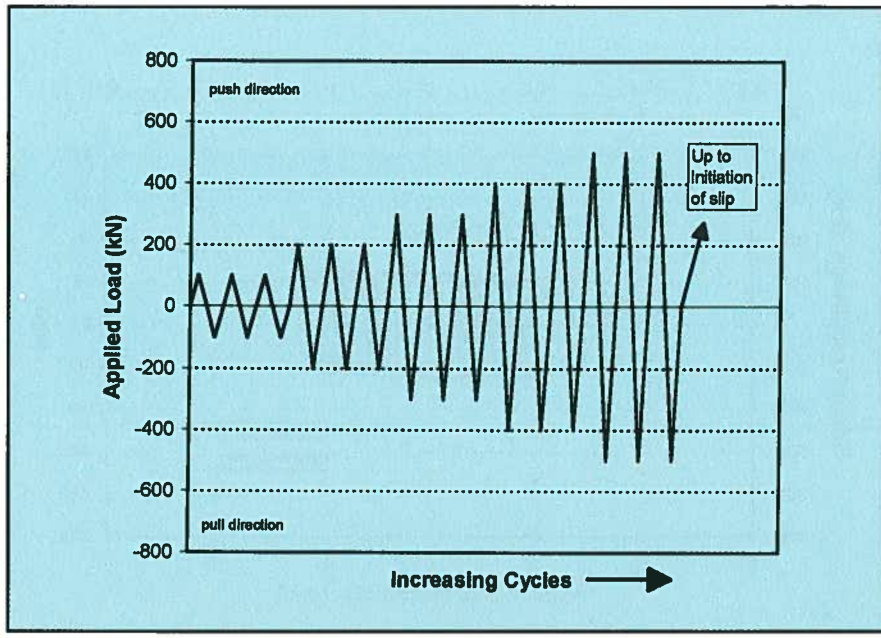
For all tested connections, the initial response was extremely stiff and elastic up to the initiation of the first slip at the interface of the drypack grout and the concrete panel. The initiation of slip occurred gradually for all connection configurations with the exception of the connection with drypack grout only, Connection DP.

Following the initiation of slip, the behavior of the drypack grout connection, Connection DP, and the post-tensioned strand connection, Connection PTS, was consistent, resulting in almost rectangular shear resistance-slip hysteresis loops, as shown in Figs. 5(a) and 5(d). The connection with mild steel continuity bars, Connection RW, exhibited parallelogram-shaped hysteresis loops with increasing shear resistance at larger slip magnitudes, as shown in Fig. 5(c). The behavior of the post-tensioned bar connection, Connection PTB, produced parallelogram-shaped hysteresis loops with decreasing shear resistance as the applied slip magnitude was increased, as shown in Fig. 5(e).

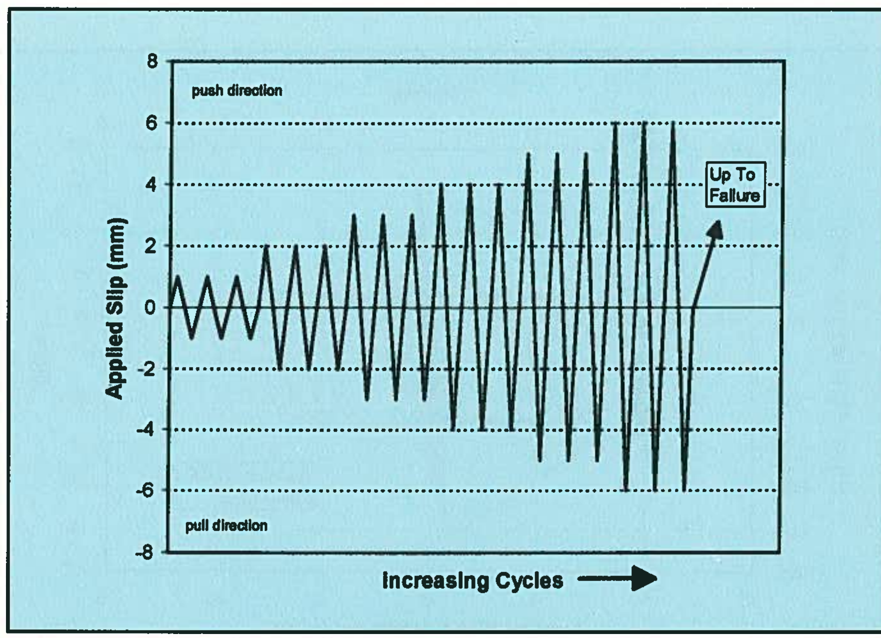
The behavior of the shear key connection, Connection SK, was somewhat different, as is evident in Fig. 5(b). Initiation of slip occurred at a load level of 400 kN (50 kips). The connection resisted a maximum applied load of 850 kN (191 kips) with a limited slip of less than 1 mm (0.04 in.). This maximum capacity was considerably higher than the maximum measured capacity for all of the tested plain surface connection configurations.

Failure of Connection SK occurred suddenly due to simultaneous cracking through the drypack grout in each of the five shear keys. This resulted in an immediate loss of shear resistance as shown in Fig. 5(b). Following failure of the drypack grout within the shear keys, a slip interface developed along the length of the connection and the behavior stabilized, producing almost rectangular shear resistance-slip hysteresis loops similar to the plain surface connections.

The behavior of the post-tensioned bar connection tested under static loading, Connection PTB-S, is shown in Fig. 5(f). Slip of the connection initiated at approximately the same load level as Connection PTB, tested under cyclic loading conditions. However, under static loading, the shear resistance steadily increased as the applied slip magnitude increased.



(a) Load control



(b) Slip control

Fig. 4. Applied reversed cyclic loading history.

The trend of increasing shear resistance of Connection PTB-S continued up to a very significant slip in excess of 35 mm (1.4 in.). At this stage, there was no measurable reduction in the thickness of the drypack grout. This behavior is consistent with behavior reported in the previous studies under monotonic loading[11-13] and is quite different from the behavior of the same connection configuration under reversed cyclic load (Connection PTB).

Failure Modes

The mode of failure for all of the connections tested under cyclic loading was due to sudden crushing and spalling of the drypack grout accompanied by a significant reduction in the thickness of the grout. The correlation between crushing of the drypack

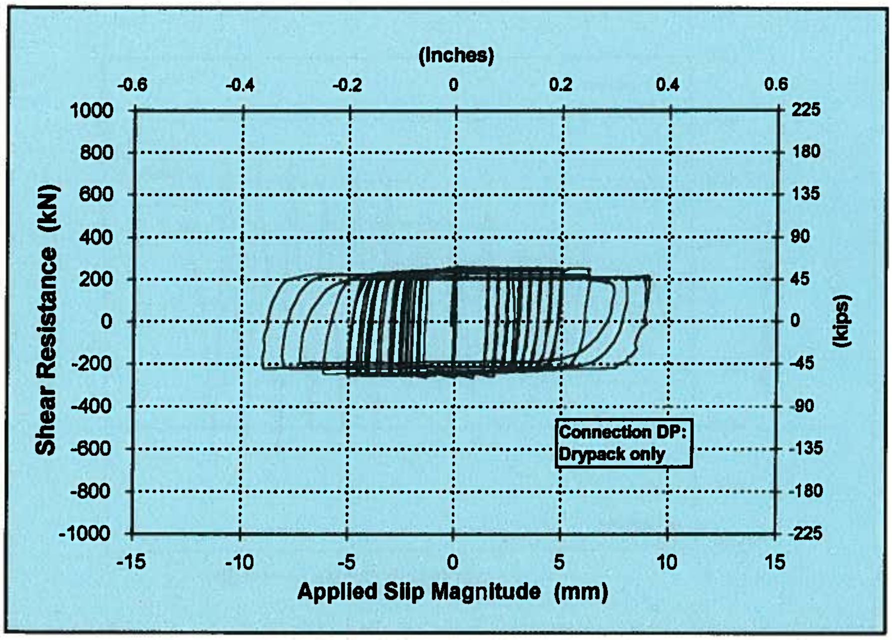


Fig. 5(a). Hysteresis loops of shear resistance vs. slip behavior for Connection DP.

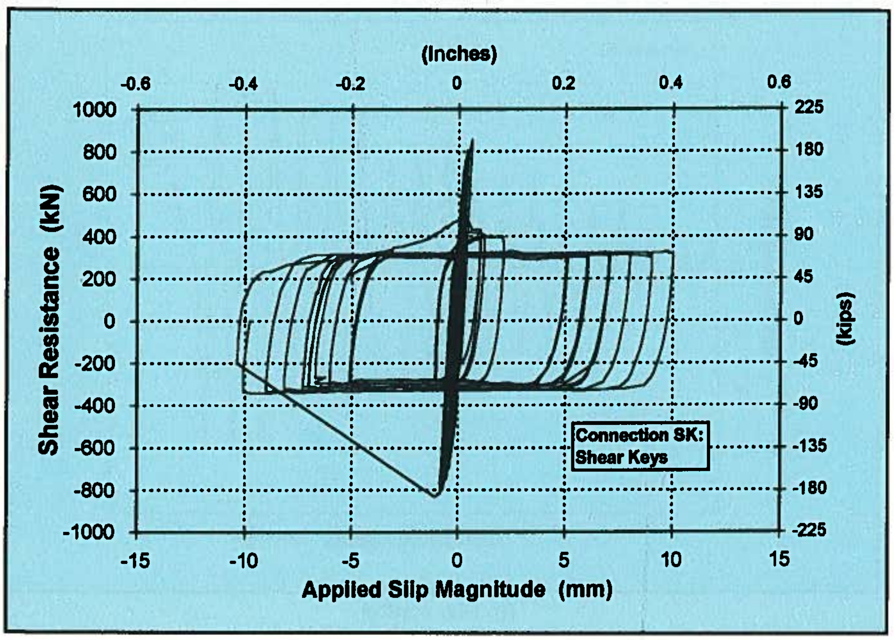


Fig. 5(b). Hysteresis loops of shear resistance vs. slip behavior for Connection SK.

grout and sudden loss of shear resistance is shown in Fig. 6. The results shown in this figure are from the connection with post-tensioned strands, Connection PTS, and are representative of the behavior observed for other connections. Prior to any notable loss of shear resistance, the reduction in grout thickness was insignificant [less than 1 mm (0.04 in.)]. After failure of the drypack grout, the hysteresis loops stabilized at a reduced shear resistance and the drypack grout was virtually ground into powder.

The effect of this occurrence on the behavior of the connection depends on the resistance mechanisms involved. While Connection DP experienced only a minor loss of shear resistance, the other configurations displayed a strength reduction in the order of 50 percent, as shown in Figs. 5(a) through (e).

For Connection DP, crushing of the drypack grout represents a reduction in the coefficient of friction. Because this is the only resistance mechanism for this configuration, the reduction in shear resistance was not substantial.

For the connection with mild steel continuity bars, Connection RW, crushing of the drypack caused transfer of the applied load normal to the connection to the two continuity bars. This behavior was confirmed by the out-of-plane buckling of one of the bars at the connection level at the end of the test. Because the vertical load was no longer acting entirely on the interface surface of the connection, shear resistance provided by friction was also reduced. The connection condition after crushing of drypack is shown in Fig. 7(a).

The post-tensioned connections, PTB and PTS, displayed a considerable loss of shear resistance upon crushing of the grout. As in the case of Connection DP, the coefficient of friction is reduced. In addition, the crushing of the grout and subsequent reduction in the grout thickness could induce a substantial loss of the prestressing force. Therefore, both the coefficient of friction and the equivalent normal force on the connection due to post-tensioning are reduced, resulting in a substantial loss of shear resistance. The condition of the connection after crushing of drypack for Connection PTB is shown in Fig. 7(b).

For Connection SK, the shear resistance mechanism before failure of the drypack is largely provided by direct bearing within the shear keys. The destruction of this resistance mechanism occurred suddenly for Connection SK due to simultaneous cracking of the drypack in all the shear keys, as shown in Fig. 7(c). This resulted in an immediate drop in the shear resistance, as illustrated in Fig. 5(b). Following failure of the drypack within the shear keys, a slip interface developed along the connection length as shown in Fig. 7(d). The behavior stabilized with almost rectangular shear-slip hysteresis loops.

DISCUSSION OF TEST RESULTS

The test results are compared using the envelope of the shear resistance-slip cyclic response. The results of Connection DP are scaled to allow direct comparison with all other specimens tested at a stress of 2 MPa (290 psi) normal to the connection.

General Behavior

The test results suggest that the cyclic shear behavior of the connections could be described in three distinct stages. The first stage describes the perfectly elastic, stiff behavior prior to initiation of slip between the drypack grout and the panels.

The second stage describes inelastic behavior after the initiation of slip. During this stage, the drypack grout remains intact and some increase or decrease in shear resistance may occur as the applied slip magnitude is increased. The shear key connection, Connection SK, exhibited a dramatic increase in shear resistance with limited slip during this stage due to the presence of the shear keys.

The third stage of behavior is initiated by sudden and extensive crushing and spalling of the drypack grout, accompanied by a significant reduction in shear resistance. This generalized behavior is illustrated in Fig. 8.

Effect of Cyclic Load

Deterioration of the drypack grout due to cyclic loading introduces an additional limit state beyond that observed under monotonic loading conditions. Initially, the grinding action of the cyclic loading caused a gradual reduction in the drypack grout thickness, producing increased compression forces in the continuity bars (Connection RW) and a loss of prestress in the specimen with post-tensioned bars (Connection PTB). At failure, cyclic loading caused significant crushing and spalling of the drypack grout. This behavior, described previously as Stage III of cyclic behavior, did not occur under monotonic loading.[11-14] The cyclic loading effect is evident by comparing companion prestressed bar connections tested under static and cyclic loading conditions, as shown in Fig. 9.

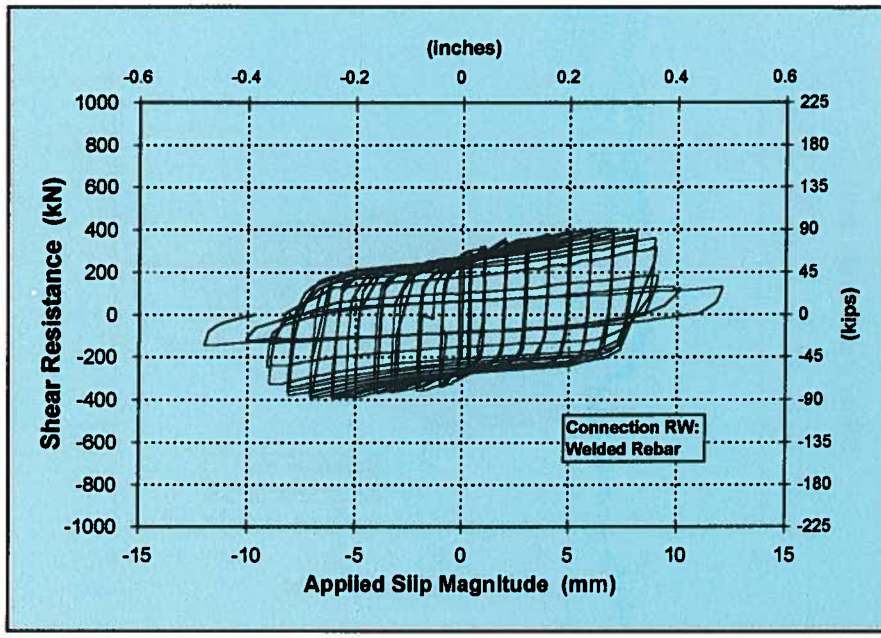


Fig. 5(c). Hysteresis loops of shear resistance vs. slip behavior for Connection RW.

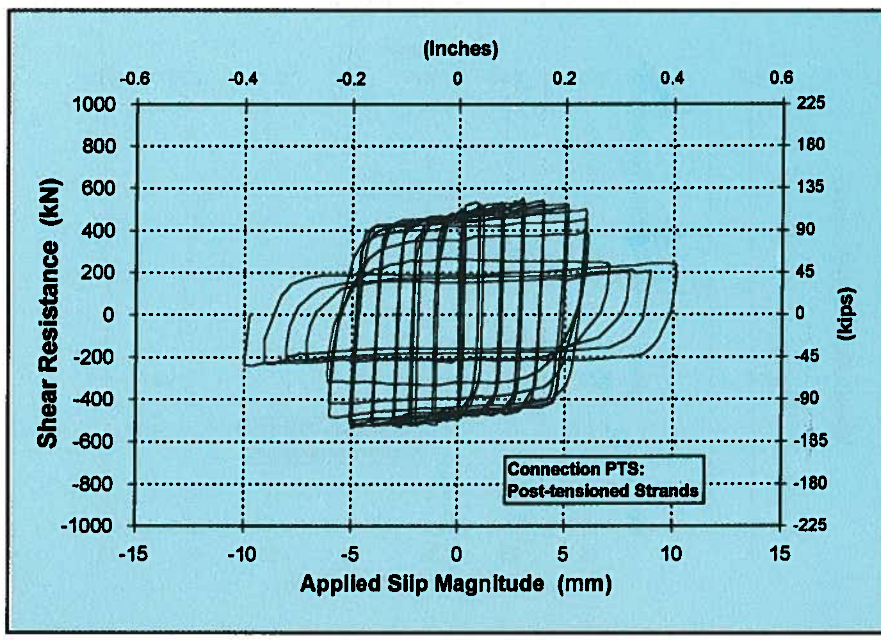


Fig. 5(d). Hysteresis loops of shear resistance vs. slip behavior for Connection PTS.

Effect of Mild Steel Reinforcement

The presence of mild steel continuity reinforcement enhanced the shear resistance relative to the connection with drypack grout only (Connection DP). The resistance of the mild steel reinforcement to deformation provides an additional component of shear resistance due to the dowel action of the reinforcing bars. This mechanism is directly related to the magnitude of the slip. As the slip is increased, the local deformation of the bars and, therefore, the resistance of the bars became more significant, increasing the shear resistance of the connection. The increased shear resistance provided by the mild steel continuity reinforcement is evident in Fig. 10 where envelopes of cyclic response for Connections DP and RW are compared.

Effect of Prestressed Reinforcement

Prestressing of the connection using bars or strands enhances the frictional resistance by increasing the effective stress normal to the connection. As a result, the overall shear resistance is increased, as is evident by comparing the shear resistance vs. slip envelopes for the prestressed connections (Connections PTB and PTS) and the drypack grout only connection (Connection DP) in Fig. 11. At failure,

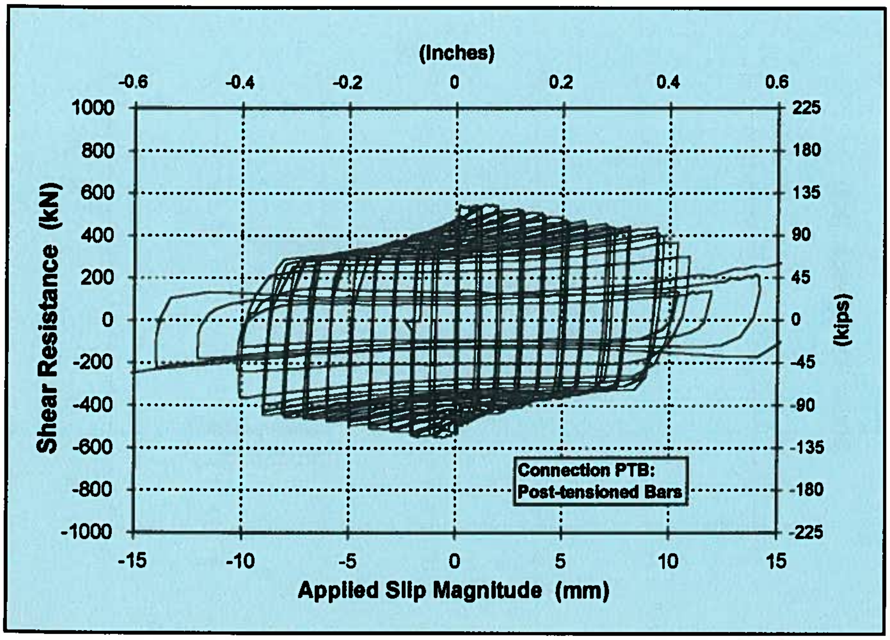


Fig. 5(e). Hysteresis loops of shear resistance vs. slip behavior for Connection PTB.

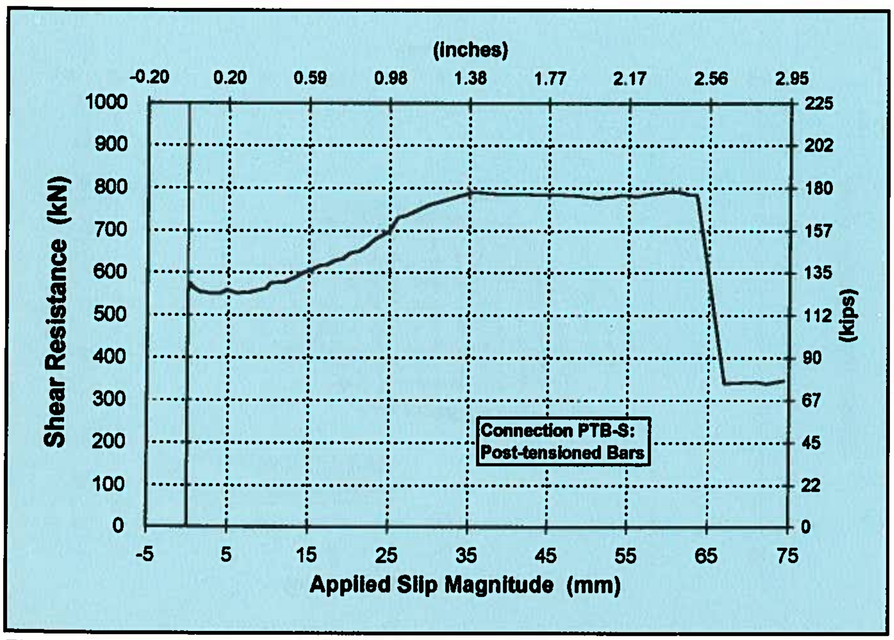


Fig. 5(f). Shear resistance vs. slip static behavior for Connection PTB-S.

significant crushing of the drypack grout could lead to a total loss of prestressing. Consequently, the additional frictional resistance due to post-tensioning will be eliminated, resulting in a shear resistance similar to Connection DP, as shown in Fig. 11.

Effect of Shear Keys

The presence of multiple shear keys significantly enhances the shear-slip resistance. This is illustrated by comparing the response of the shear key connection, Connections SK and DP, in Fig. 12. The maximum shear resistance of Connection SK was more than three times that of Connection DP. The increase in the shear resistance of Connection SK is mainly due to the transfer of shear by direct bearing of the drypack grout within the shear keys.

Prior to cracking of the drypack grout within the shear keys, the slip was limited to less than 1 mm (0.04 in.). After cracking of the drypack grout within the shear keys, slip occurred along the length of the connection and the shear resistance was provided mainly by friction.

The ultimate shear resistance of Connection SK was 40 percent higher than Connection DP. The higher shear resistance of Connection SK could be attributed to the different slip interfaces for the two connection types at ultimate. The slip interface for Connection SK is enhanced due to confinement of the drypack grout within the shear keys. It is assumed that this interface, consisting of a combination of confined drypack-to-drypack regions within the shear keys and concrete-to-drypack regions elsewhere, will provide a higher frictional resistance in comparison to the interface for Connection DP, consisting of smooth concrete-to-drypack regions only.

SHEAR RESISTANCE MECHANISMS

On the basis of the observed experimental behavior and the measured results, mechanisms of shear transfer were proposed for each connection configuration. Where possible, mathematical models were proposed for the prediction of the connection capacity at the various limit states of cyclic behavior. However, it should be mentioned that no attempt was made in the models to evaluate the slip magnitude for the corresponding limit state. The models presented predict the shear strength of the connection at various limit states.

Connection With Mild Steel Reinforcement

The shear resistance mechanism for plain surface connections with mild steel continuity reinforcement is provided by two components: (1) friction at the drypack grout-to-panel interface and (2) resistance of the continuity bars to deformation. The initial frictional resistance, V_{f1}, is provided by the net gravity stress acting on the connection, σ_{n1}, as shown in Fig. 13. The remainder of the gravity load is resisted by the continuity reinforcement, F_b.

The shear resistance provided by the continuity bars does not become active until slip has occurred. Contribution of the bars to the shear resistance was in a progression of a flexural and kinking mechanism. The shear resistance of the bar due to the flexural mechanism, V_{h1}, shown in Fig. 14(a), is dominant until the formation of plastic hinges in the bars. The shear resistance of the bars after the formation of plastic hinges is due to a kinking mechanism, as shown in Fig. 14(b).

As horizontal slip is increased, a tensile force is induced in the bars along the kinked length. The vertical component of this force exerts a vertical stress on the drypack grout and produces additional frictional resistance V_{f2}. This is shown in Fig. 14(b) and is referred to as clamping action. In addition, the horizontal component of the tensile force in the bar, V_{h2}, will directly resist the applied shear load.

The proposed shear resistance mechanisms are primarily a function of the force within the continuity bars. Prediction of the response based on such a mechanism requires the prediction of the force in the continuity bar at different slip magnitudes. Because the variation of the compressive force in the bar cannot be rationally predicted, the approach becomes very complicated and inconvenient for design purposes. Detailed information is given in Ref. 14.

To avoid the necessity for prediction of the bar forces, the following simplification may be applied. Earlier, it was stipulated that the grinding action of the reversed cyclic loading causes a reduction in the thickness of the connection and consequently induces compression strains in the continuity bars. Due to the induced precompression, the bar will not yield in tension under cyclic load as was observed under monotonic loading conditions.[11] Experimentally measured strains obtained for Connection RW under cyclic loading indicated that the maximum tension strain increment in the bars due to the kinking mechanism was limited to 40 percent of the yield level.

Thus, for design purposes, the increased frictional resistance due to clamping action, V_{f2}, may be based on the assumption of a maximum tensile stress increase in the bars equal to 40 percent of the yield strength. It should

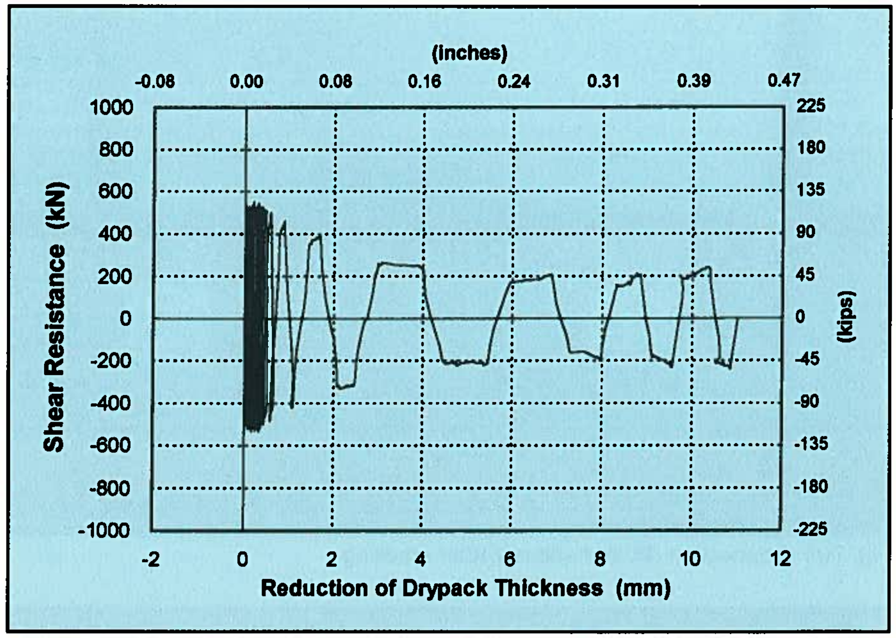


Fig. 6. Correlation between loss of shear resistance and crushing of drypack grout.

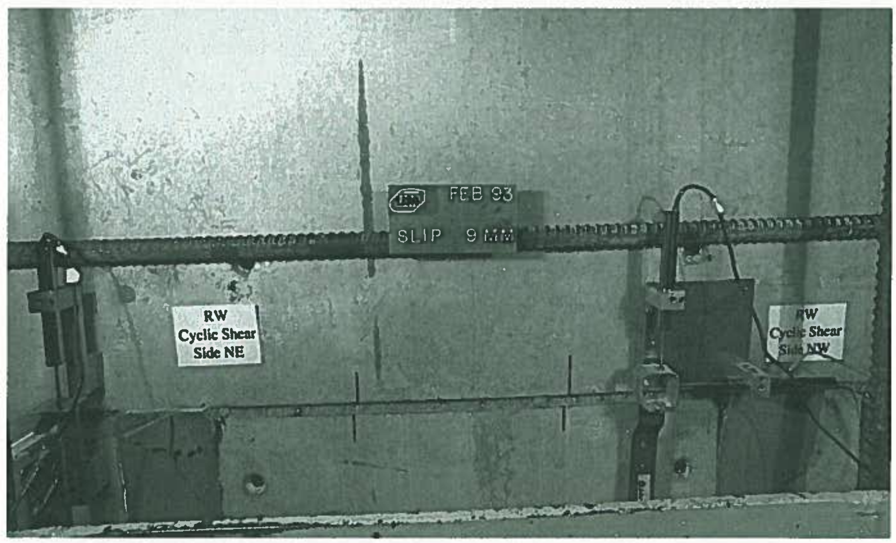


Fig. 7(a). Failure mode for Connection RW.

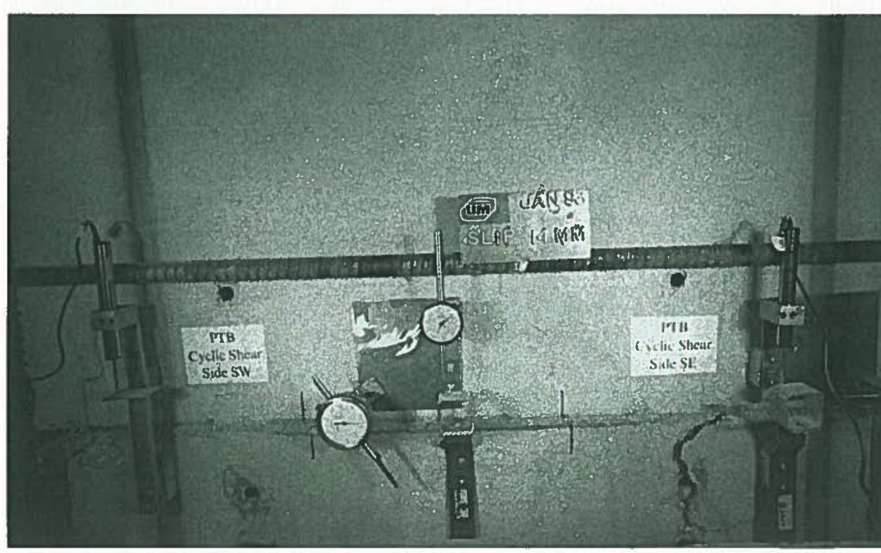


Fig. 7(b). Failure mode for Connection PTB.

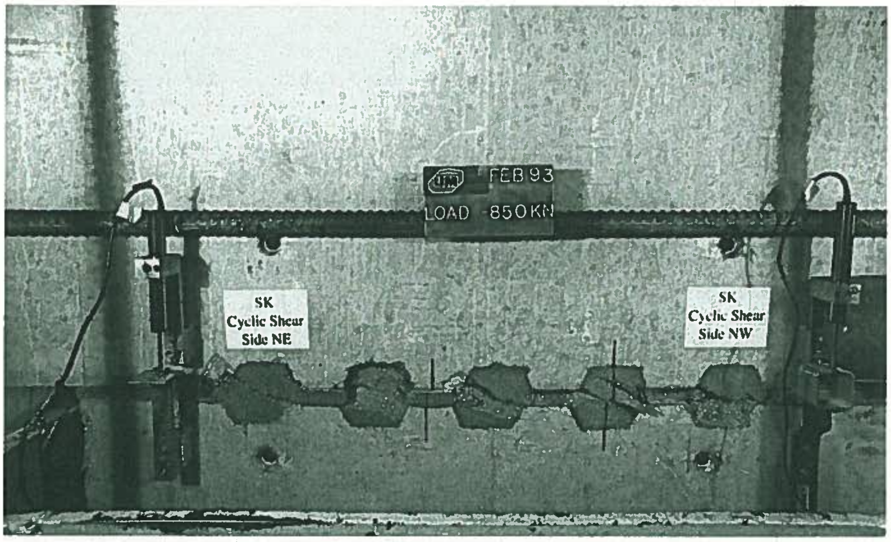


Fig. 7(c). Connection SK immediately after cracking.

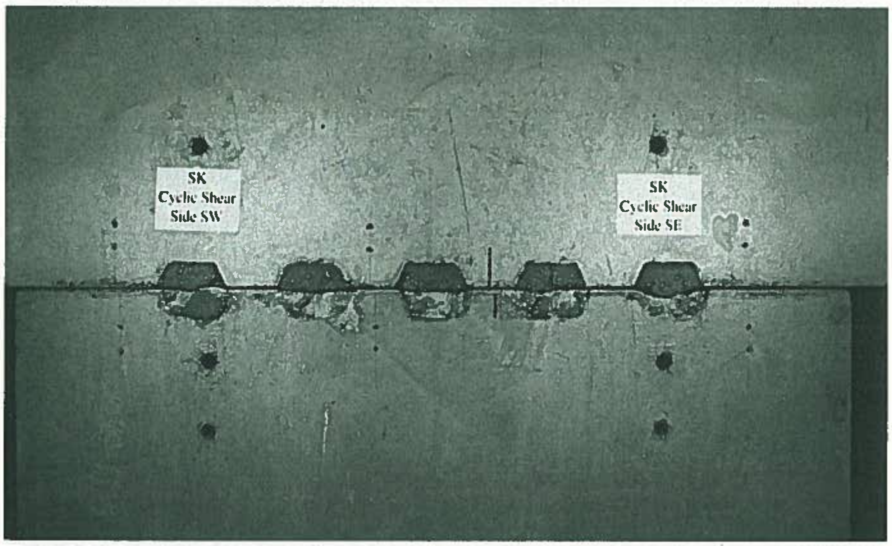


Fig. 7(d). Connection SK at failure.

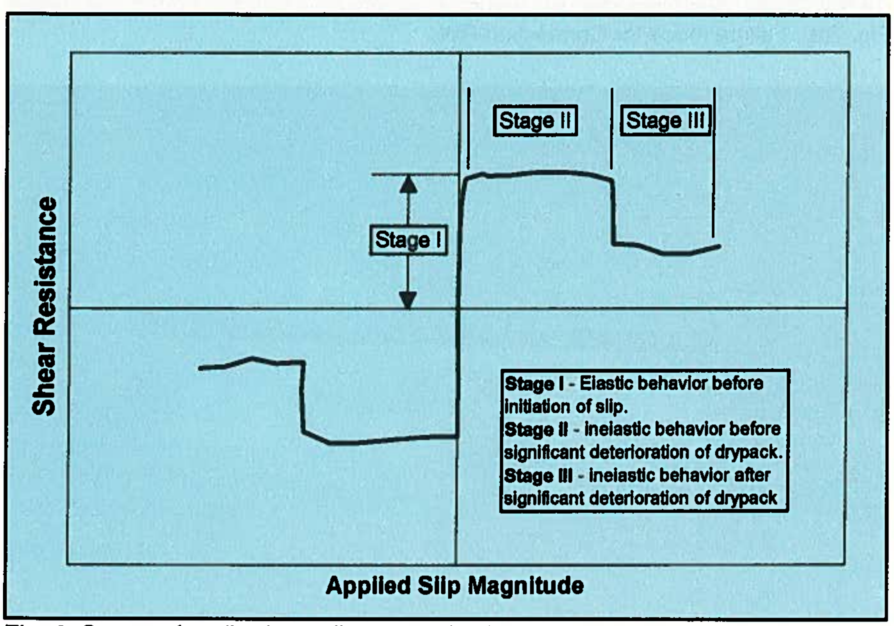


Fig. 8. Stages of cyclic shear-slip connection behavior.

be noted that this value may be dependent on the percentage of steel across the connection region. Although the horizontal component, V_{h2}, can also be calculated in this manner, examination of the experimental results suggests that the contribution of this mechanism was negligible.

For design purposes, prediction of the initiation of slip and the maximum and ultimate shear resistance of the connection may be achieved using the general form given as follows:

$$V_r = V_n + V_b \quad (1)$$

where V_n is the frictional resistance provided by load normal to the connection given by:

$$V_n = \mu \sigma_n A_c \quad (2)$$

and V_b is the frictional resistance provided by the clamping action of the bar:

$$V_b = \mu A_s f_s \quad (3)$$

Therefore:

$$V_r = \mu(\sigma_n A_c + A_s f_s) \quad (4)$$

where

μ = coefficient of friction
= 0.80 initially to 0.6 at ultimate (based on test results)

σ_n = compressive stress due to normal loads

A_c = cross-sectional area of connection

A_s = total cross-sectional area of continuity bars

f_s = tensile stress increase in continuity bar ($< f_y$)

Prior to initiation of slip, the component of shear resistance due to clamping action of the bar is not activated because it is dependent on the extent of slip. At this stage, the shear resistance is provided only by interface friction due to gravity loads and may be predicted using Eq. (2) with $\mu = 0.8$.

The maximum shear resistance of the connection is developed after considerable slip along the connection. Shear resistance is provided by friction due to gravity loads and clamping action resulting from deformation of the bars. As described previously, the maximum tensile stress increase in the continuity bars due to kinking action, f_s, may be taken as 40 percent of the yield strength of the bar. At this stage, the drypack grout remains intact and the shear resistance may be predicted using Eq. (4) with $\mu = 0.8$.

At ultimate, extensive crushing of the drypack grout occurs and the clamping action of the continuity reinforcement is no longer reliable due to possible buckling of the bars at the connection level. In addition, the coefficient of friction is reduced due to crushing of the drypack grout. At this stage, the shear resistance of the connection is provided only by friction due to gravity loads and may be predicted using Eq. (2) with $\mu = 0.6$.

Connection With Prestressed Reinforcement

The shear resistance mechanism of plain surface connections with post-tensioning is mainly provided by friction at the drypack grout-to-panel interface. The frictional resistance of the connection is due in combination to the applied vertical gravity loads and post-tensioning stresses normal to the connection. The total shear resistance could be separated into two components to illustrate the contribution of the applied load normal to the connection and the prestressing force:

$$V_r = V_n + V_p \tag{5}$$

where V_n is the frictional resistance provided by the load normal to the connection given by:

$$V_n = \mu \, \sigma_n A_c \tag{6}$$

and V_p is the frictional resistance provided by post-tensioning:

$$V_p = \mu \, \sigma_p A_c \tag{7}$$

Therefore:

$$V_r = \mu \, (\sigma_n + \sigma_p) \, A_c \tag{8}$$

where

μ = coefficient of friction
= 0.80 initially to 0.6 at ultimate (based on test results)

σ_n = compressive stress due to normal loads

σ_p = compressive stress due to post-tensioning

A_c = cross-sectional area of connection

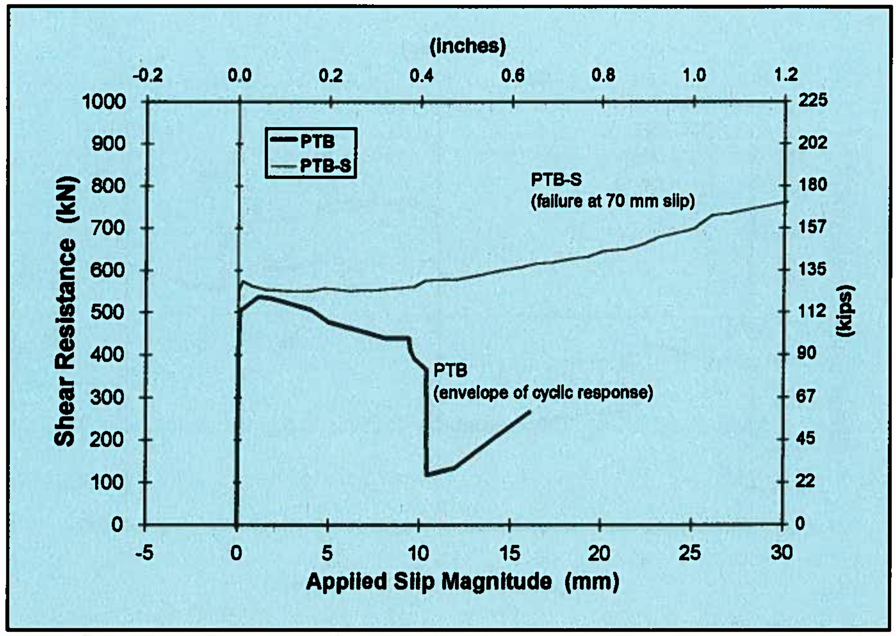


Fig. 9. Effect of cyclic load.

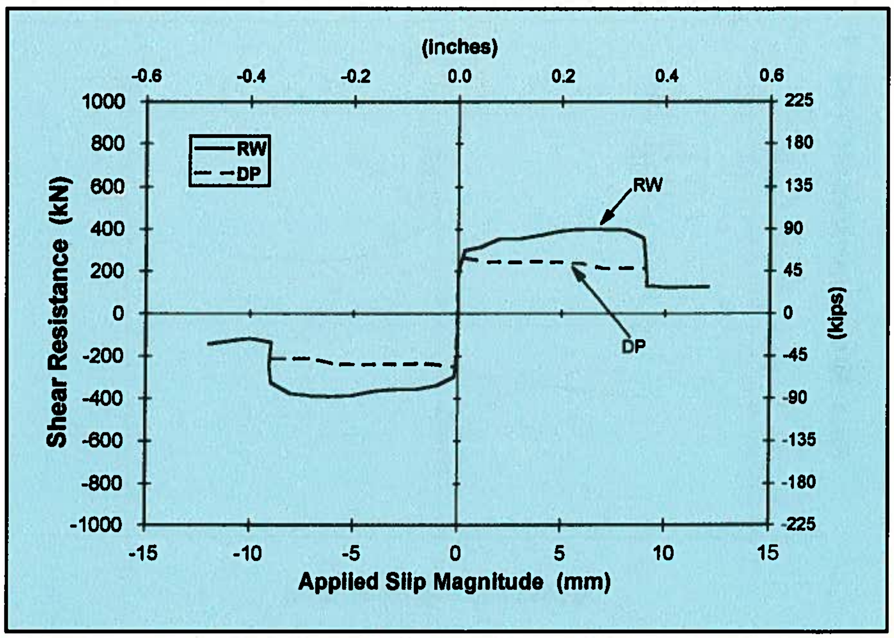


Fig. 10. Effect of mild steel continuity reinforcement.

For both types of post-tensioned connections investigated in this experimental program, the shear resistance at the initiation of slip and the maximum shear resistance were very similar. At both of these limit states, the drypack grout is still intact and the loss of prestressing is insignificant. Thus, the initiation of slip and the maximum shear resistance of the connection may be predicted using Eq. (8) with $\mu = 0.8$.

At ultimate, extensive crushing and spalling of the drypack grout was observed for both connection types with post-tensioning. At this limit state, crushing of the drypack grout was accompanied by a significant reduction in the thickness of the connection, leading to a complete loss of prestressing. As a result, the frictional resistance provided by post-tensioning is reduced to zero. Crushing of the drypack grout also lowered the coefficient of friction of the connection. Thus, the ultimate shear resistance of the connection may be predicted using Eq. (6) with $\mu = 0.6$.

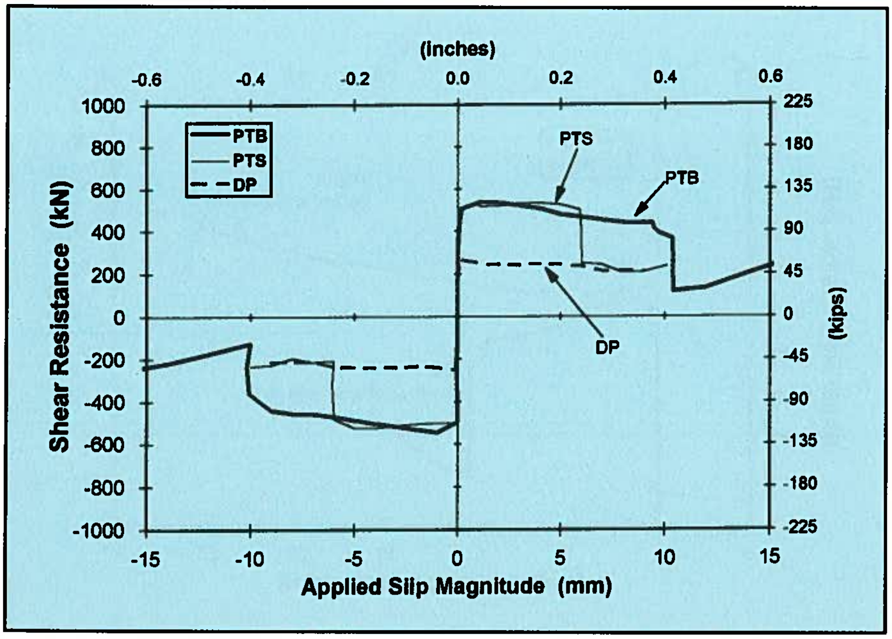


Fig. 11. Effect of post-tensioning.

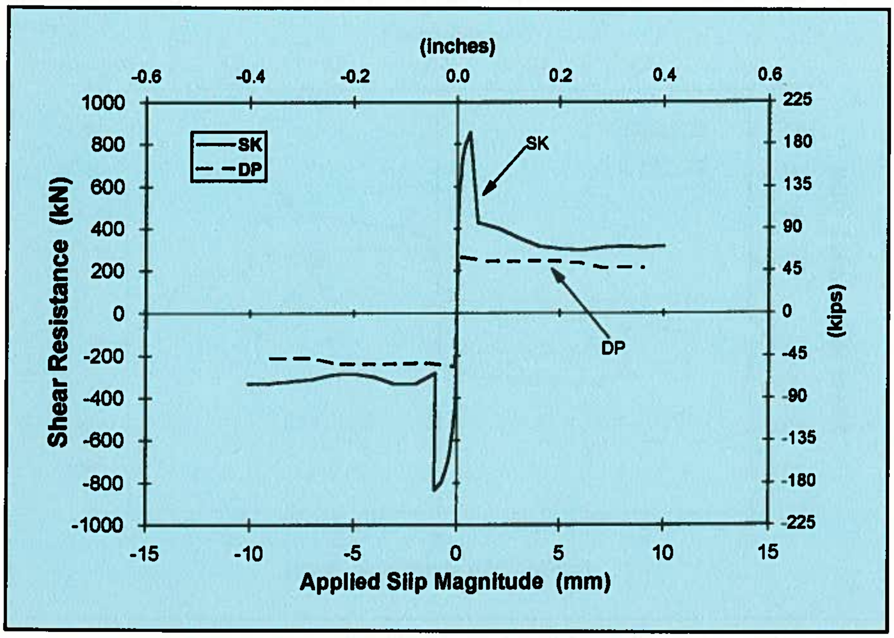


Fig. 12. Effect of shear keys.

Connection With Multiple Shear Keys

The observed cyclic behavior of the connection with drypacked multiple shear keys was not significantly different from the behavior observed previously under monotonic loading conditions.[11,13] The mathematical models proposed by Serrette et al.[13] for monotonic loading conditions appear to also be applicable under cyclic loading. The reader is referred to Ref. 13 for a complete description of these models.

DESIGN EXAMPLES

Consider a typical connection for a precast concrete loadbearing interior shear wall panel at the base of a 10-story building. The connection is 1200 mm (48 in.) long and 150 mm (6 in.) wide with a 20 mm (3/4 in.) drypacked thickness between the two precast panels. The gravity load on the connection from the upper stories is equivalent to a stress, σ_n, of 2 MPa (290 psi). Vertical continuity between the two panels is provided by two continuity reinforcement at 900 mm (36 in.) on center.

Two examples are included to show the prediction of the shear strength for connections with mild steel and prestressed reinforcement. The continuity reinforcement is: (1) 25M (#8), Grade 400 MPa (60 ksi) mild steel reinforcing bars and (2) 12.7 mm (1/2 in.) diameter seven-wire prestressing strand, Grade 1860 (270 ksi). The characteristic properties of the different materials used in the connection are:

Concrete:

Cylinder compressive strength,
$f_c' = 48$ MPa (6.96 ksi)

Drypack grout:

Cube compressive strength,
$f_g'' = 55$ MPa (7.98 ksi)

Cylinder compressive strength,
$f_g' = 0.85 f_g'' = 46$ MPa (6.67 ksi)

Mild Steel Bars:

Area of one bar,
$A_s = 500$ mm^2 (0.79 sq in.)

Yield strength,
$f_y = 400$ MPa (60 ksi)

Ultimate strength,
$f_u = 680$ MPa (98.6 ksi)

Prestressing Strand:

Area of one strand,
$A_{ps} = 99$ mm^2 (0.153 sq in.)

Yield strength,
$f_{py} = 0.90 f_{pu} = 1675$ MPa (243 ksi)

Ultimate strength,
$f_{pu} = 1860$ MPa (270 ksi)

Example 1: Mild Steel Reinforcement Connection

Initially, the component of shear resistance due to the bars is zero because the slip is zero. The shear resistance at the initiation of slip is provided by friction and is calculated using Eq. (2):

$$\begin{aligned} V_r &= \mu \sigma_n A_c \\ &= 0.8(2)(180{,}000) \\ &= 288 \text{ kN } (64.8 \text{ kips}) \end{aligned}$$

As discussed previously, at maximum shear resistance, the tensile stress increase in the bars is limited to 40 percent of the yield strength. Therefore, the maximum shear resistance can be determined as follows:

$$\begin{aligned} V_r &= \mu (\sigma_n A_c + A_s f_s) \\ &= 0.80[(2)(180{,}000) \end{aligned}$$

$$+ (2)(500)(0.4 \times 400)]$$
$$= 416 \text{ kN } (93.5 \text{ kips})$$

At failure, the severe drypack grout deterioration causes a reduction in the coefficient of friction from 0.8 to 0.6. The bars buckle out-of-plane and their contribution to the shear resistance reduces to zero at this stage. The shear resistance becomes:

$$V_u = \mu(\sigma_n A_c + A_s f_s)$$
$$= 0.60\,[(2)(180{,}000) + (2)(500)(0)]$$
$$= 216 \text{ kN } (48.6 \text{ kips})$$

The predicted maximum and minimum strength levels are compared to the envelope of the measured cyclic shear-slip behavior of mild steel connection, Connection RW, with identical configuration to the example in Fig. 15. The measured resistance at initiation of slip for Connection RW was within 1 percent of the predicted value. The maximum resistance was 4 percent lower than the predicted value and the measured ultimate strength was 14 percent lower than the predicted value.

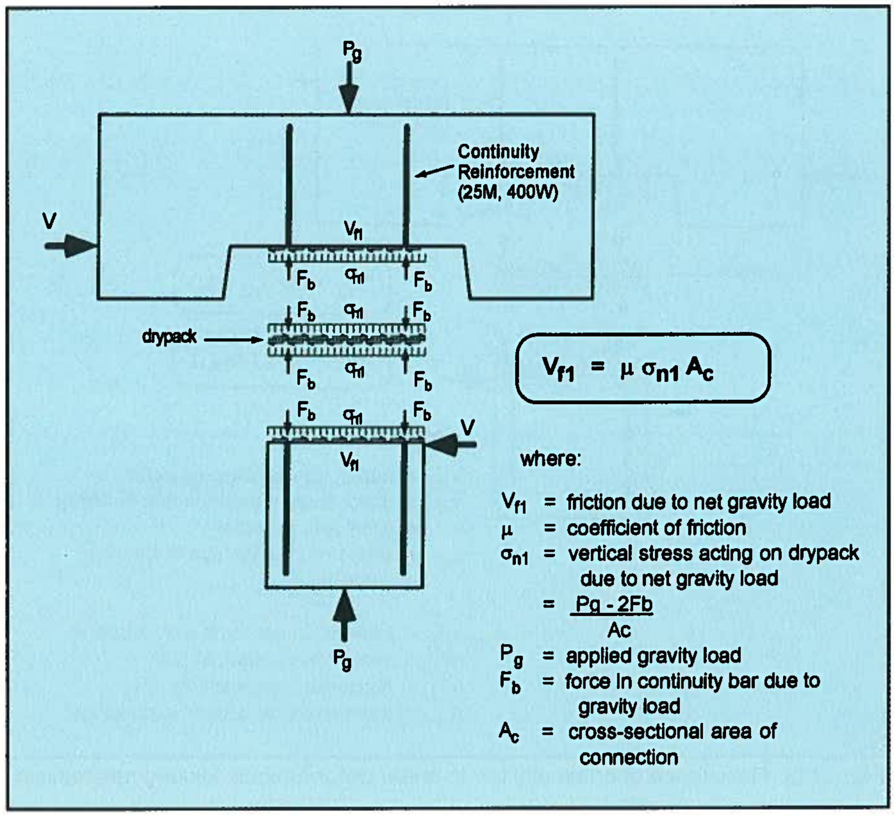


Fig. 13. Initial frictional resistance provided by net gravity load acting on Connection RW.

Example 2: Prestressed Connection

The initiation of slip and the maximum shear resistance of a connection post-tensioned using strands can be predicted using Eq. (8) with a nominal coefficient of friction of $\mu = 0.80$. The shear resistance is determined as follows:

$$V_r = \mu(\sigma_n + \sigma_p)A_c$$
$$= 0.80(2 + 1.2)(180{,}000)$$
$$= 460 \text{ kN } (103.5 \text{ kips})$$

At failure, the severe drypack grout deterioration caused a reduction in the connection thickness and a complete loss of prestressing. As a result, the contribution from component V_p is reduced to zero and the shear resistance is provided only by friction due to the load normal to connection, V_n, with a reduced coefficient of friction of μ = 0.6. Thus, the shear resistance becomes:

$$V_u = \mu(\sigma_n + \sigma_p)A_c$$
$$= 0.60(2 + 0)\,(180{,}000)$$
$$= 216 \text{ kN } (48.6 \text{ kips})$$

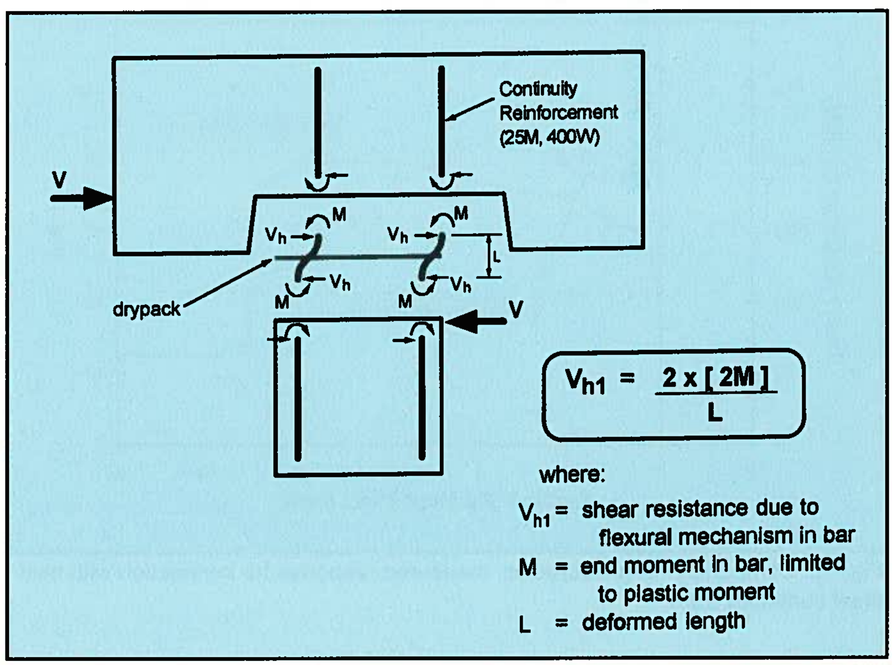


Fig. 14(a). Resistance of continuity bar to shear deformations: flexural mechanism.

The predicted maximum and minimum strength levels are compared to the envelope of the measured cyclic shear-slip behavior for the connection post-tensioned using strands, Connection PTS in Fig. 16. This connection has an identical configuration to the example. The measured maximum strength was 15 percent higher than the predicted value and the measured ultimate strength was 4 percent higher than the predicted value.

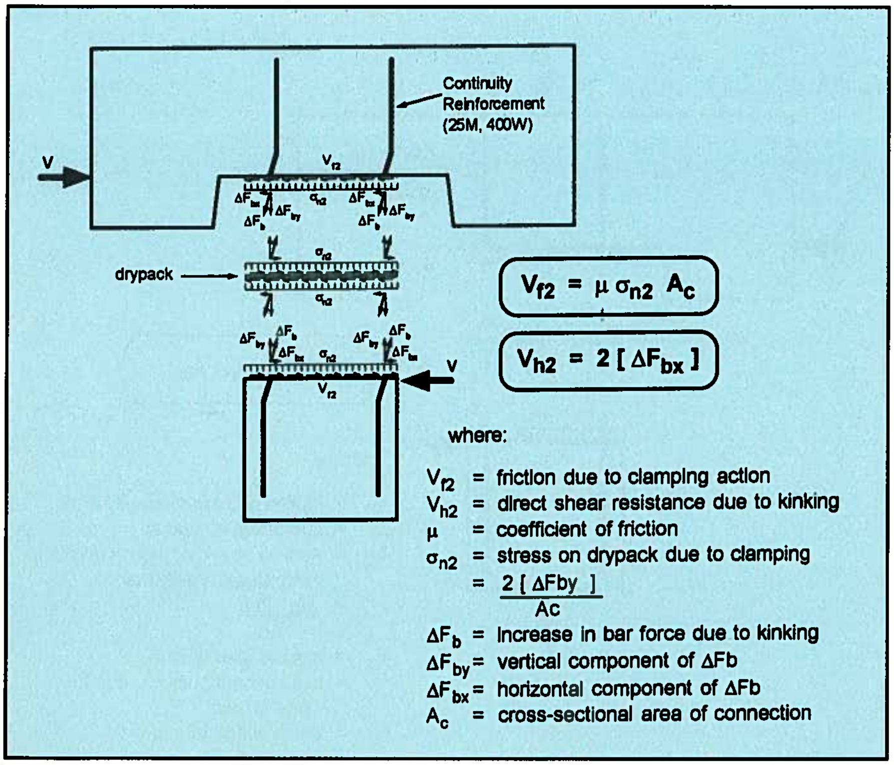


Fig. 14(b). Resistance of continuity bar to shear deformations: kinking mechanism.

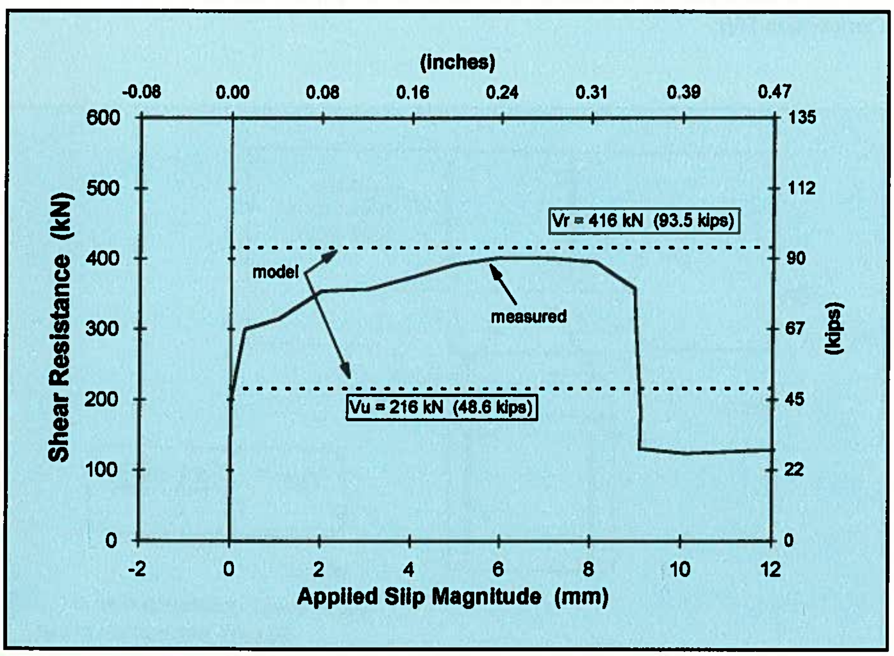


Fig. 15. Comparison of predicted vs. measured response for connection with mild steel continuity bars.

CONCLUSIONS

Based on the results of this study, the following conclusions can be drawn:

1. The cyclic behavior of the connection could be identified by three distinct limit states: (a) elastic and very stiff behavior prior to initiation of slip; (b) inelastic behavior with stable hysteresis before significant joint deterioration; and (c) failure with significant joint deterioration and more than 20 percent reduction in the shear resistance.

2. The mode of failure for all connection configurations tested under cyclic loading was due to significant crushing and spalling of drypack grout. This introduced an additional limit state beyond that observed for identical specimens tested under monotonic loading conditions. In all cases, failure occurred after a considerable number of fully reversed cycles and slip magnitudes larger than 5 mm (0.2 in.).

3. The shear resistance forms an elasto-plastic, unconfined mechanism with irrecoverable slip. The slip could only be recovered if the loading direction is completely reversed.

4. The shear resistance for connections with mild steel continuity reinforcement is mainly provided by two components: (a) interface friction and (b) resistance of the continuity bar to deformation. A model based on these mechanisms was introduced and agrees well with experimental results.

5. The shear resistance of connections with post-tensioning, using either strands or bars, is mainly provided by friction at the drypack grout-to-panel interface. Using prestressing increases the vertical stress on the connection, thus increasing the overall frictional resistance. A model was introduced that agrees well with experimental results at the limit states of cyclic behavior.

6. The cyclic behavior of connections with shear keys can be predicted using models proposed previously for monotonic loading conditions.

RECOMMENDATIONS

1. The findings of this research[14] and the companion programs[8-13] have defined the behavior and capacity of typical and new horizontal connections for precast concrete loadbearing shear wall panel systems. These results are a significant step towards development and acceptance of precast concrete wall panel systems in seismic zones.

2. Typical connection details currently used in practice for precast wall panels exhibit stable hysteretic behavior under reversed cyclic shear loading. All connection configurations investigated in this program exhibited sustained shear capacity for many cycles of loading and slip magnitudes in excess of 5 mm (0.2 in.).

3. The use of grouted shear keys significantly increased the shear resistance of the connection and limited horizontal slip. Both of these attributes are desirable to overall seismic performance of this structural system. Therefore, the use of shear keys is recommended in precast wall panel structures in seismic zones.

4. The elasto-plastic behavior of the shear resistance mechanism provides a source of energy dissipation. However, the slip mechanism is not recoverable, and accumulation of slip over the height of the structure could lead to structural instability. Therefore, if it is desired to utilize horizontal slip as an energy dissipation mechanism, the slip must be constrained to avoid instability.

5. Further research is needed to define the response of the structural system under earthquake conditions. Using the connection behavior defined in these research programs,[8-14] analytical and/or experimental studies of structural response must be performed to ensure overall satisfactory seismic performance.

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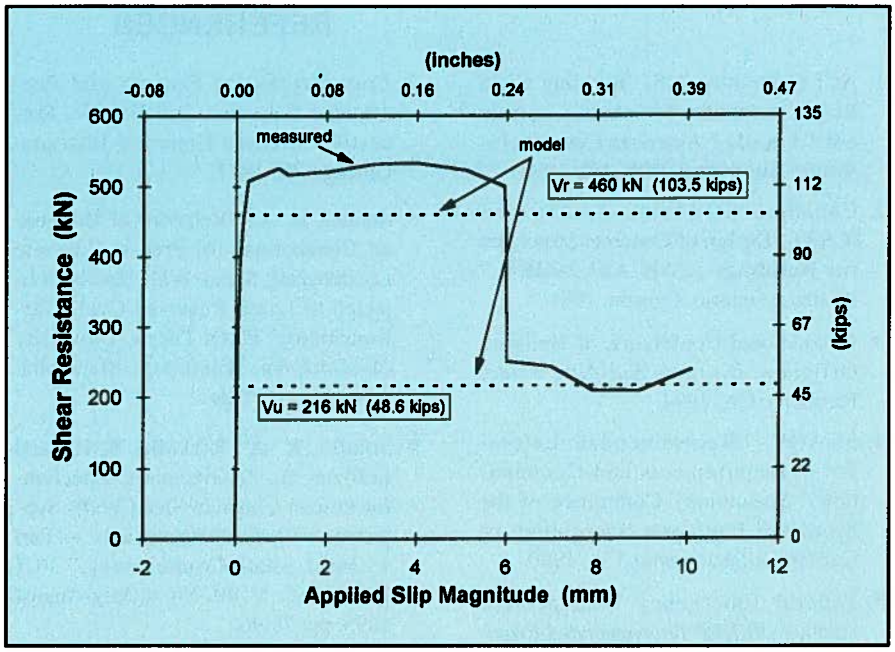

Fig. 16. Comparison of predicted vs. measured response for connection with post-tensioned strands

APPENDIX — NOTATION

A_c = total cross-sectional area of connection

A_s = total cross-sectional area of continuity bars crossing connection

f_c' = cylinder compressive strength of panel concrete

f_g' = equivalent standard cylinder compressive strength of drypack grout

f_g'' = cube compressive strength of drypack grout

f_s = tensile stress increase in mild steel continuity bars

f_y = nominal yield strength of mild steel continuity bars

F_b = compressive force in mild steel continuity bars due to applied gravity load

V_b = frictional resistance provided by clamping action of bar

V_{f1} = shear resistance component provided by friction due to net gravity load acting on drypack

V_{f2} = shear resistance component provided by friction due to clamping action (kinking mechanism)

V_{h1} = shear resistance component provided by flexural mechanism in continuity bars

V_{h2} = direct shear resistance component provided by kinking mechanism

V_n = frictional resistance provided by load normal to connection

V_p = frictional resistance provided by post-tensioning

V_r = shear resistance of connection (for design)

V_u = ultimate shear resistance of connection after failure of drypack grout

μ = coefficient of friction

σ_n = compressive stress on connection due to normal loads

σ_{n1} = net compressive stress normal to connection due to gravity loads

σ_{n2} = compressive stress normal to connection due to clamping action

σ_p = compressive stress normal to connection due to post-tensioning

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